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SULPHITE RAILROAD BRIDGE

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Introduction

Objectives

The objective of this engineering study was to examine and describe the structural behavior of the Sulphite Railroad Bridge, built in 1896 by the Boston & Maine Railroad. The Sulphite Bridge is a rare surviving example of a wooden Pratt deck truss.

Two mathematical models of the Sulphite Bridge were designed and used for analyses–one to model the prestressed version of the bridge, and the other for the unprestressed form–to develop a clearer understanding of the Pratt form and its behavior under several possible conditions. These involved dead load analyses and live load analysis using contemporary Boston & Maine locomotive and railcar weights. The live load analysis incorporated the unique manner that trains impose live loads on bridges. The effects of time-dependent behavior of wood on the Sulphite Bridge were also studied. As a whole, these analyses demonstrate the process that was probably used to design the bridge.

Background and engineering evolution of the Pratt truss

Thomas and Caleb Pratt’s truss, like the slightly earlier Long and Howe designs, was a truss form that utilized prestressing to optimize performance, but it accomplished prestressing in a different manner. Prestressing is the mechanical introduction of loads into truss members in addition to the dead loads resulting from the weight of the truss. Joints between wooden members generally perform better under compression than tension. Ideally, prestressing ensures that these joints remain in compression under all load conditions.

Where Long used wedges driven into wooden joints to induce compression forces, Howe and Pratt both employed iron rods in place of wood for certain members that normally were in tension. While iron rods were virtually useless in compression because they would bend easily, they handled tensile forces very well. Threaded nuts at their ends allowed builders to tighten them like a clamp to apply compression forces on adjacent wooden members and joints.

Howe's design used iron in place of wood for the vertical posts, and tension here compressed the joints between its wooden diagonals and chords. The Pratt truss, which put iron diagonal members in tension, was more sophisticated, and it ultimately proved to be an excellent form for all-metal bridges as well. Figure 1 shows drawings from the Pratt patent of 1844, including the parallel-chord version as well as an alternative form with an arched upper chord.

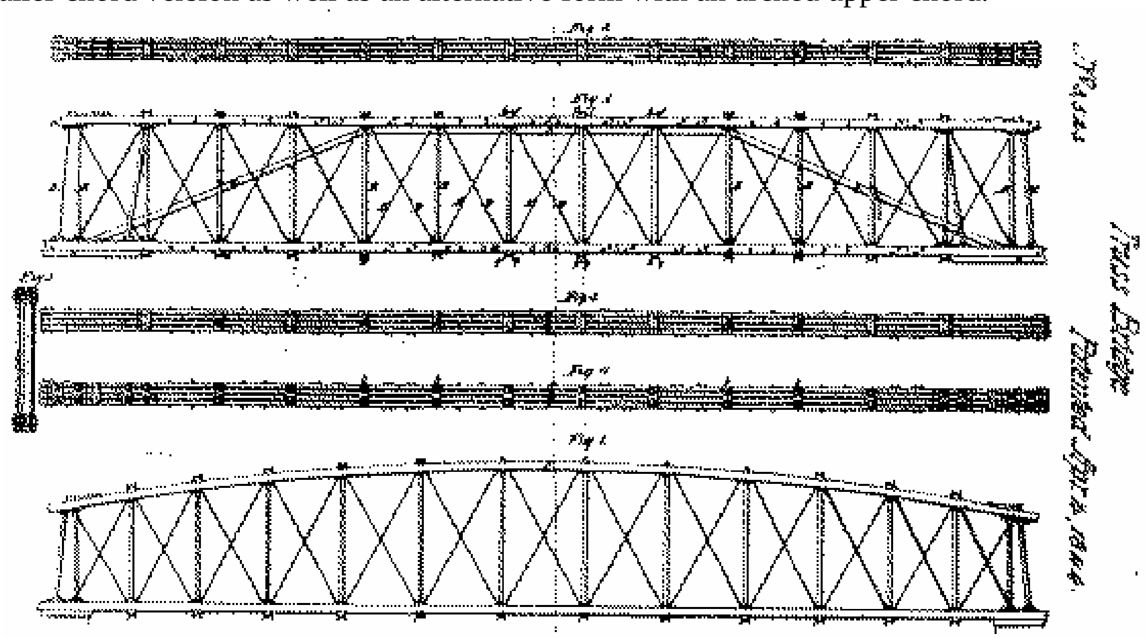


Figure 1. Diagrams from Pratt's 1844 bridge patent[1]

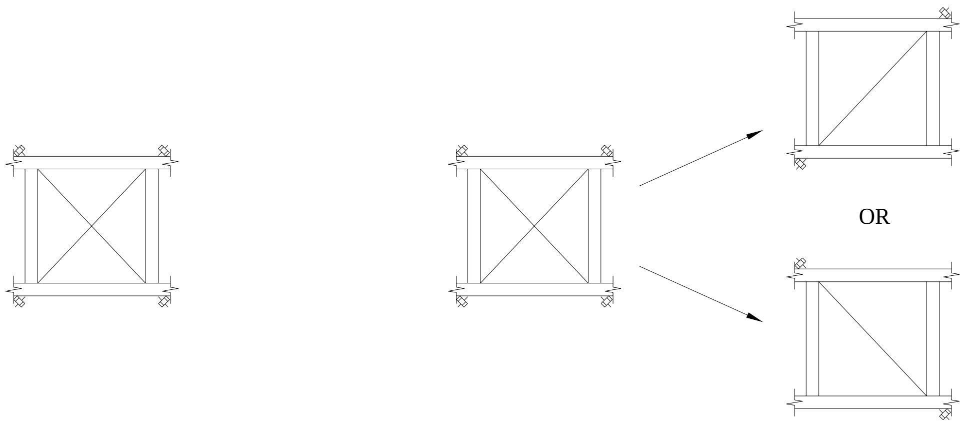
The Pratts' patent described pairs of slender, tension-only iron diagonals in combination with other wood members. In the claims they state, "The bracing by means of tension bars extending diagonally across each panel of a bridge truss has long been known and used; but the system of bracing and counter bracing by means of tension bars crossing each other in each panel, is believed to be new …."[2] Clearly, the Pratts intended both diagonals to be active in tension, which was possible because they were pretensioned by tightening the nuts. While defining the concept, they did not describe how to size or proportion the pairs of diagonals, nor the procedure for tightening them.

The Pratts understood that the diagonals pointing downward toward the center of the span would normally be in tension under both dead and live loads, but they had concerns that some live-load conditions, particularly as a heavy vehicle moved across the bridge, might put some diagonals in compression. With easily bendable iron rods, that kind of stress reversal could cause a complete collapse, so they included iron counter diagonals (or simply counters) in their early designs. If a main diagonal ever encountered a stress reversal, the situation would simultaneously increase the tension in the counter and maintain the truss panel's integrity.

The Pratts originally intended for both slender elements to be active in tension as represented by Figure 2a, even though a thorough analysis of such a statically indeterminate form was not yet

[1] United States Patent No. 3523, granted to Thos. W. Pratt and Caleb Pratt, 1844.
[2] United States Patent No. 3523.

possible. By the 1880s, advances in structural mechanics led design engineers to favor statically determinate forms having only one diagonal active for any live load condition. With one diagonal carrying the resultant shear force in the panel by tension, the other diagonal could be considered to have buckled and, thus, not be playing an active role. The change in view on the role of counters is depicted graphically in Figure 2.



a) Prestressed, slender diagonal members active for all live load conditions

a) Unprestressed, slender diagonal members either but not both active for any particular live load condition

Figure 2. Change in the role of counter diagonals in a Pratt truss

While engineers widely adopted the analytical method as a practical tool, uncertainty and disagreement about the exact mechanism and role of counters persisted. Mansfield Merriman and Henry S. Jacoby, who wrote the most widely-used structural analysis book of the 1890s, claimed, "The main and counter brace cannot both be strained at the same time by any system of loading."[3] However, with some inconsistency, they later stated, "In trusses whose diagonals take only tension, the counter ties are adjustable in order to be drawn up to a certain degree of tension when the bridge is unloaded. The stresses thus induced in these truss members is called initial tension, and serves to prevent the vibration of the diagonals in the counter panels under moving loads and to stiffen the truss as a whole."[4]

A. Jay Du Bois, whose textbook was also well known, stated, "The strain in a counter-brace is, therefore, due entirely to the action of the live load. The dead load causes no strain in it whatever. The main braces, therefore, in any case, are those braces which are called into action by the dead load. The counter-braces, those which are called into action by the live load only."[5] In the following paragraph, however, he also commented, "By properly screwing up the counters … the girder may be held down to that deflection which would be caused by the live load when it covers the whole span, and the girder thus rendered very rigid. The live load as it comes on would then act simply to relieve the strains in the counters without adding anything to those

[3] Mansfield Merriman and Henry S. Jacoby, *Roofs and Bridges. Part II. Graphic Statics* (New York: John Wiley & Sons, 1890), pg. 75.
[4] Merriman and Jacoby, *Graphic Statics*, pg. 85
[5] A. Jay Du Bois, *The Strains in Framed Structures* (New York: John Wiley & Sons, 1890), pg. 57

existing in the braces themselves. Under such circumstances, all the pieces sustain always a heavy strain, except the counter-braces, and in these the strain, though fluctuating in amount, is always the same in character."[6] This statement was not entirely correct, and it revealed an incomplete understanding of the behavior of prestressed counters.

Kunz provided a third statement on the use of counters in his 1915 book: "If the diagonals can only take tension (eyebar diagonals), counter diagonals are provided in all those panels in which the main diagonals would be in compression when the live load counteracts the tension from dead load. When the counter-diagonal acts it has a compression stress from the dead load equal to the dead-load tension stress in the main diagonal. The maximum live-load tension in the counter-diagonal is equal to the maximum live-load compression which could occur in the main diagonal if there were no counter-diagonal."[7]

Kunz's statement describes a rational approach to designing nonprestressed "counters." It is likely that in 1896 the designers of the Sulphite Railroad Bridge adopted such an approach rather than the Pratts' concept of having both diagonals prestressed and actively sharing the applied load. They were not alone, as the concept and practice of "screwing up" (prestressing) counter diagonals slowly fell into disuse by the turn of the twentieth century.

Railroad bridge design

Railroad trains, being both heavy and long, impose live loads on a bridge that are quite different from those imposed by individual vehicles. A string of cars will put something approaching a uniformly distributed load on the truss, but a locomotive leading a train across a bridge results in a combination of the fairly concentrated load of the locomotive and the distributed load from the following cars. The proportion of all this changes as the train moves across the span. Additionally, steam locomotives imposed some dynamic, vertical loads due to some inherently unbalanced forces that were an unavoidable result of their drive system.

As locomotives and trains grew increasingly heavier during the second half of the nineteenth century, bridge engineers had to develop ways to be sure their designs accounted for all of these loads with an adequate margin of safety. Fortunately, the era saw major developments in structural mechanics and materials technology. During this period, engineers attained a good understanding of the behavior of statically determinate structures and the mathematical means to analyze them.

A principal change during this period was in the size and power of locomotives; from 1873 to 1911, there was a four-fold growth in locomotive weight.[8] Locomotive weight, particularly when they were "double-headed" on heavy trains, became the dominant factor in bridge design.

Henry S. Jacoby summarized the impact of these increases in the design live loads used for railroad bridges in 1902:

> The form of loading for bridges almost universally specified by railroads of the United States consist of two consolidation–eight-coupled–locomotives

[6] Du Bois, pg. 57

[7] F.C. Kunz, *Design of Steel Bridges, Theory and Practice* (London: McGraw-Hill Book Company, Inc., 1915), pg. 79.

[8] Kunz, see Table 4, pg. 14.

> followed by a uniform train load. These loads are frequently chosen somewhat larger than those that are likely to be actually used for some years in advance; but sometimes the heaviest type of locomotive in use is adopted as the standard loading. Of the railroads whose lengths exceed 100 miles, located in the United States, Canada and Mexico, only two out of 77 specified uniform train loads exceeding 4000 lb. per lineal foot of track in 1893; while in 1901 only 13 out of 103 railroads specified similar loads less than 4000 lb. In 1893, 37 railroads specified loads of 3000 lb., and 29 of 4000 lb.; while in 1901, 4000 lb. was specified by 50, 4500 lb. by 14, and 5000 lb. by 17 railroads. The maximum uniform load rose from 4200 lb. in 1893 to 6600 lb. in 1901.
>
> In a similar manner in 1893 only one railroad in 75 specified a load on each driving-wheel axle exceeding 40,000 lb.; while in 1901 only 13 railroads out of 92 specified less than this load. In 1893 only 21 of the 77 railroads specified similar loads exceeding 30,000 lb. The maximum load on each driving-wheel axle rose from 44,000 lb. in 1893 to 60,000 lb. in 1901.[9]

The process of designing and procuring bridges by railroad companies also evolved. Initially, bridge companies designed, fabricated, and erected bridges for clients using proprietary methods. This system evolved into one where the railroads or their consulting engineers defined specifications for their bridges. The design, fabrication, and erection were then either procured through competitive bidding, or, for smaller bridges, done "in-house."

The development of specifications was partially in response to bridge failures—notably the Ashtabula Bridge collapse in December of 1876 and the Firth of Tay Bridge collapse in 1879—that demonstrated the increasing need to have fully competent engineers do the actual design work. This method did, however, depend on specifications that accurately represented the actual service conditions expected. The development of such specifications was a long and involved process that included incremental developments spanning almost a century, beginning at least as early as those contained in Stephen Long's 1830 booklet about his truss and its construction.[10]

By far the most important specifications were those of Theodore Cooper, which were first defined for the Erie Railway in 1879 and revised in 1888 and 1906.[11] Cooper's specifications had numerous provisions. One of the most important was the specification for the live load expected from a train. F.C. Kunz in *Design of Steel Bridges, Theory and Practice*, listed Cooper's "E-50," "E-55," and "E-60" live loads, showing that, in 1915, the Boston and Maine Railroad, builders of the Sulphite Railroad Bridge, used Cooper's E-50 loading.[12]

Using these specifications, engineers had to perform a series of laborious steps to design each member of the bridge:

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[9] "Bridge Building and Bridge Works in the United States, No. V, Contracts ad Specifications," *The Engineer* 95 (February 20, 1903): pg. 190.

[10] D.A. Gasparini and K. Geraci, "Development of Structural Design Specifications in the Nineteenth Century," Presented at the 12th International Bridge Conference, Pittsburgh, PA, June 1995.

[11] Gasparini and Geraci.

[12] Kunz, Table 2, pg. 11.

a) Determine the position of the set of axle loads that produces the maximum axial force in the member,
b) Calculate the maximum force,
c) Size and detail the member to safely carry the maximum axial force.

To cope with the continuingly variable nature of the live loads imposed by moving trains, this process utilized the concept of *influence lines*, which are explained in the analyses sections of this report. Note, too, that any difference in the actual weight of a member from the engineer's initial assumption would change the dead load it imposed on the other members, often requiring that they be re-evaluated. It was an iterative process that could be quite involved.

The Sulphite Railroad Bridge

The Sulphite Railroad Bridge, sometimes referred to as the "upside-down covered bridge," is a rare example of a deck-type covered bridge having its truss structure below the floor, or deck. This Pratt truss bridge was used to transport sulphur ore to a nearby pulp and paper factory. The Boston and Maine Railroad built it and operated it until 1973.

Despite being burned by vandals in 1980, the structural integrity of the Sulphite Bridge remains good. The siding has been completely burned away, as well as the deck along the bottom (Figure 3), but the three 60'-long spans are still standing. The chords and vertical posts, while badly charred, are still present, and the stone piers are in good shape (Figure 4). The bridge does not have any significant downward sag as a result of the damage.

Figure 3. Sulphite Railroad Bridge, view inside truss. Field photograph.

Figure 4. View of pier. Field photograph.

The top and bottom chords of the bridge are each comprised of three parallel wooden members, spaced with wooden blocks, which allow the diagonal tension rods to pass through, and tied with iron bolts (Figure 5). Vertical members are placed between bearing plates on each of the chords. Cross bracing is placed through the otherwise open space between the trusses, as well as between the two top chords and two bottom chords (Figure 6). In addition to cross-braces, there are also tie rods with turnbuckles between the chords. Essentially, the lateral bracing systems of the top and bottom chords are prestressed Howe trusses.

Figure 5. View of bottom chords, showing spacer blocks and bolt. Field photograph.

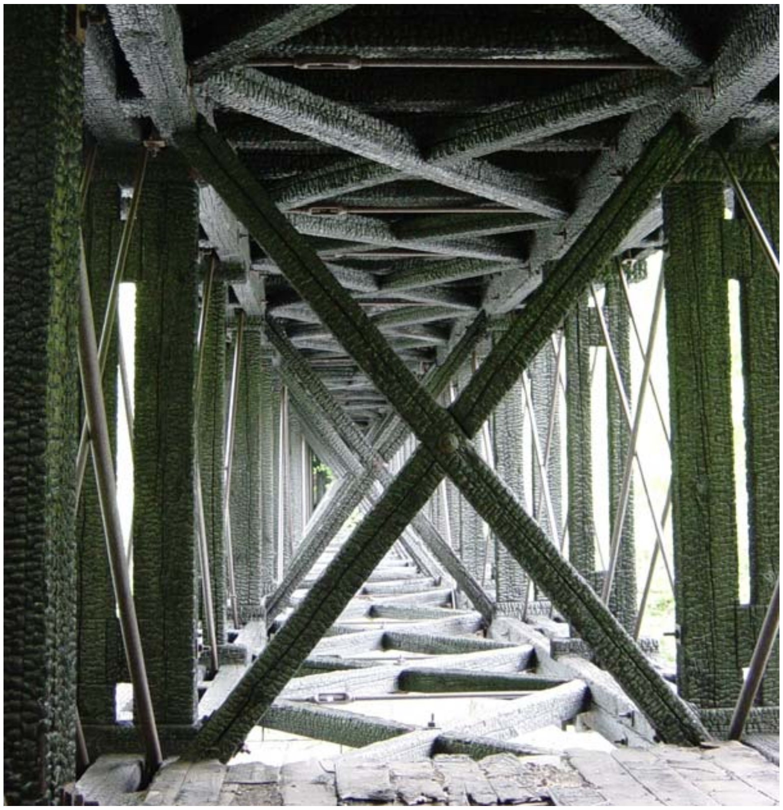

Figure 6. View inside Sulphite Bridge, showing Howe lateral bracing trusses at deck and roof level. Field photograph.

The castings that anchor the diagonal tension members (Figure 7) of the Sulphite Bridge are one of its most interesting features. These large iron pieces, carefully cast in a shop, bear against the outsides of the top and bottom chords where the diagonal rods frame in, and are held in place by the forces in the diagonals.

Figure 7. Connection casting for diagonal rods at bottom chord. Field photograph.

Figure 7 shows how the bearing surfaces and washers for the iron rods on the nodal castings are circular in order to minimize bending in the rods. The castings have “keys” at the front and back that penetrate into the chords and transfer horizontal forces. The surface area of the castings in contact with the wood is very large, thus minimizing compression and creep perpendicular to the grain of the wood.

Since the railroad track is on top of the Sulphite Bridge, the construction of the roof and upper chords must be sufficient to transfer the live loads from the train to the bridge structure. Interestingly, the designers did not place the trusses directly under the running rails, which would have substantially reduced the loads on, and size of, the ties. As built, the ties must act as beams instead of simply transmitting the train’s load straight down to the trusses. Modern designs typically place the trusses directly under the rails, but the Sulphite Bridge’s designers may have wanted the additional width between the trusses in order to have wider lateral trusses to aid in handling lateral loads.

By 1896, the single-diagonal Pratt truss had become the most widely-used structural form for steel bridges, so it is interesting that the builders chose to use the Pratt form for this largely wood bridge. Perhaps it was simply a matter of engineering economics. If structural analyses showed that the forces in the chords and verticals were small enough so that wood was sufficient, the designer may well have been able to achieve a sizeable saving compared to an all-metal bridge.

The Sulphite Bridge is most likely an unprestressed Pratt form, with counters included only as insurance against stress reversals in the main diagonal rods. Another indication that this bridge was not prestressed is the lack of a counter diagonal in the end-span panels. Indeed, a statically determinate structural analysis of the Sulphite Bridge truss reveals that the tension rod in the last panel does not see a stress reversal. Such an analysis is clearly outlined in Merriman's book.

Structural behavior calculations

The behavior of the Sulphite Railroad Bridge was analyzed under dead load alone and under combined dead and live loading. Prestressed and unprestressed models of the bridge initially were analyzed under dead load alone to gain insight about the likelihood of the bridge being prestressed when built. The live load analysis was performed with a typical locomotive loading for the period to determine the maximum axial stresses in the truss. The counters were assumed to be inactive unless stress reversals occurred in the main diagonals. Although an accurate analysis of the effects of creep on the truss would require a model capable of performing time-dependent stress-strain calculations, the linear elastic models available for this study were used to estimate the effects of an assumed creep strain of ±0.0005.

Two plane frame models for one span of the Sulphite Bridge were analyzed with MASTAN2 structural analysis software, one model with counter braces and one without, as shown in Figures 8 and 9. The frame was modeled as simply supported, with element properties as given in Table 1. The computer model computed lengths of the diagonals from the coordinates of their end nodes, which were placed at the centroids of the chords. Since in the actual bridge the diagonal rods extend above the top chord and below the bottom chord, the assumed modulus of elasticity for the rods was reduced by 5.2 percent to compensate for the difference and appropriately model the axial stiffness of the rods.

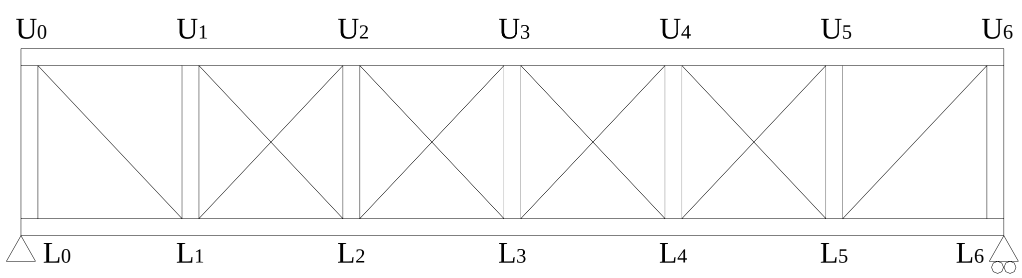

Figure 8. Plane frame model with counter braces

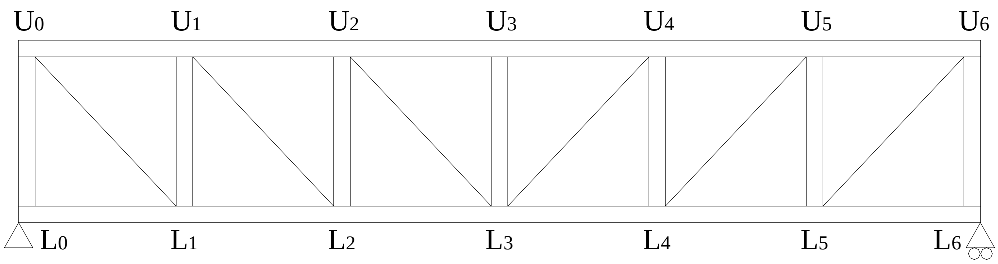

Figure 9. Plane frame model without counter braces

Table 1. Element properties for Sulphite Bridge truss (tension rod properties given per rod)[13]

Element	Length (ft)	Area (A) (in^2)	Moment of inertia (I) (in^4)	Section modulus (S) (in^3)	Elastic modulus (E) (psi)	Stiffness (k=EA/L) (lbs/in)
Top chords	720.000	234.0	3295.5	507.0	1,200,000	390000
Bottom chords	720.000	207.0	2281.2	396.8	1,200,000	345000
Verticals	92.004	215.0	1792.0	358.3	1,200,000	2804226
	92.004	200.0	1666.6	333.4	1,200,000	2608582
	92.004	72.0	216.0	72.0	1,200,000	939090
Tension rods						
(2) 2.5" Dia. + (2) 1.5" Dia.	169.92	4.909	1.917	1.534	28,000,000	808963
	169.92	1.767	0.249	0.331	28,000,000	291197
(2) 2" Dia.	169.92	3.142	0.785	0.785	28,000,000	517684
(2) 1.5" Dia.	169.92	1.767	0.249	0.331	28,000,000	291197
(2) 1.25" Dia.	169.92	1.227	0.120	0.192	28,000,000	202220
(2) 1" Dia.	169.92	0.785	0.049	0.098	28,000,000	129421

The models in Figures 8 and 9 represent two different types of behavior that can occur in a Pratt truss. The model in Figure 8 is the prestressed version, where all diagonals are assumed to be prestressed with an adequate tension force and remain active for all load cases. Figure 9 represents the situation where the truss is not prestressed, the live load is uniform, and, thus, the counters are not active. It should be noted that Figure 9 only represents only one possible mode of behavior, as defined by the active members of a truss. This prestressed six-panel truss (Figure 8) has only one mode of behavior because all members are active regardless of the location of a live load, but the unprestressed truss actually has five possible modes of behavior, as illustrated in Figure 10, depending on the location of the live load. The model of Figure 9 is probably the

[13] Based on the influence line for diagonal U_4L_3, the element should be larger than its corresponding counter diagonal, U_3L_4. According to the Hoyle, Tanner, and Associates drawing for the Sulphite Bridge, however, the counter diagonal is bigger than the main diagonal. Because these data are inconsistent with the structural analysis, it is very possible that a mistake was made in the design, or erection, or field measurements of the bridge. Based on this observation, the decision has been made to switch the member size for U_4L_3 and U_3L_4 for the purpose of obtaining consistent analysis results.

one used by the bridge's designers for their analyses, and they may have used any compressive forces in the main diagonals predicted by the model to size the corresponding counters.

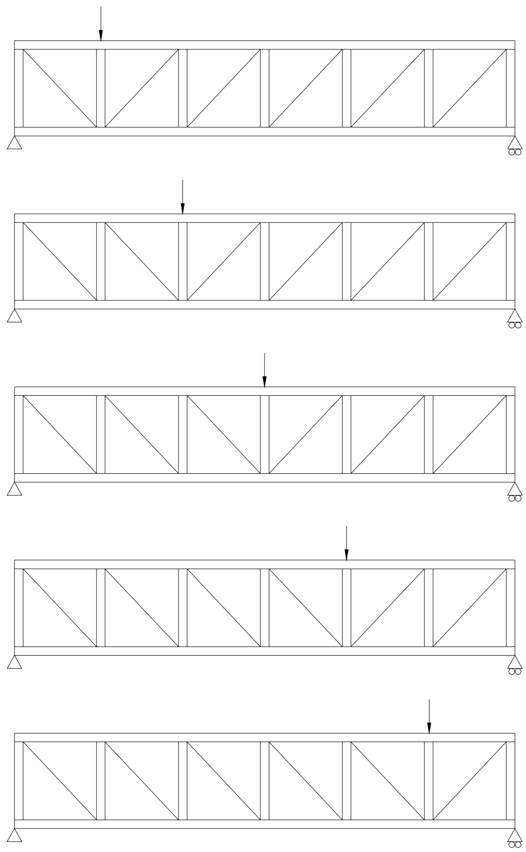

Figure 10. Possible behavior modes for the Sulphite Bridge based on location of a large live load

The bridge's dead load was calculated from its member properties. Table 2 gives a list of these members with their dimensions and weight, along with a total weight for one span of the bridge. The density of the wood, identified as a yellow pine by the U. S. Forest Products Laboratory, was assumed to be 38 lbs/ft^3, and the iron was assumed to have a density of 484 lbs/ft^3.[14] The weights of the tension rod bearing plates were estimated using the approximate dimensions shown on engineering drawings provided by Hoyle, Tanner, and Associates, Inc.[15]

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[14] USDA Forest Service, Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726-2398

[15] Hoyle, Tanner, and Associates, Inc, 150 Dow Street, Manchester, NH 03101

Table 2. Volume and weight details for one span of the Sulphite Bridge

Member	Length (ft)	Width/Diameter (ft)	Depth (ft)	Volume (cu.ft.)	Quantity	Density (lbs/ft^3)	Weight (lbs)
Truss							
Top chords	60.00	1.500	1.083	97.50	1	38.000	3705
Bottom chords	60.00	1.500	0.958	86.25	1	38.000	3278
Verticals	7.67	0.833	1.792	11.45	2	38.000	870
	7.67	0.833	1.667	10.65	2	38.000	809
	7.67	0.500	1.000	3.83	3	38.000	437
Vertical ties	0.69	0.396	0.896	0.245	14	38.000	130
Bolts	2.21	0.063	-	0.0068	14	484.000	46
Cross-bracing	10.20	0.458	0.4583	2.14	3.5	38.000	285
Tension rods							
(2) 2.5" Dia. + (2) 1.5" Dia.	14.16	0.208	-	0.483	4	484.000	934
	14.16	0.125	-	0.174	4	484.000	336
(2) 2" Dia.	14.16	0.167	-	0.309	4	484.000	598
(2) 1.5" Dia.	14.16	0.125	-	0.174	4	484.000	336
(2) 1.25" Dia.	14.16	0.104	-	0.121	4	484.000	234
(2) 1" Dia.	14.16	0.083	-	0.0772	4	484.000	150
Bolster block (bottom)				1.50	5	484.000	3640
Bolster block (top)					7	484.000	3822
Total truss weight (x2)							**24297**
Siding							
Panels	60.00	10.50	0.0729	45.94	1	38.000	1746
Nailer	60.00	0.167	0.333	3.33	4	38.000	507
Total siding weight (x2)							**4505**
Deck							
Stringers (cross-bracing)	11.08	0.125	0.542	0.750	13	38.000	371
Decking	60.00	5.708	0.115	39.24	1	38.000	1491
Ties	7.88	0.125	-	0.0966	7	484.000	327
Total deck weight							**2189**
Roof							
Cross-bracing	11.08	0.500	0.500	2.77	14	38.000	1474
Ties	7.88	0.125	-	0.0966	7	484.000	327
Girders	12.00	0.833	0.833	8.33	26	38.000	8233
Bolsters	60.00	0.833	0.333	16.67	5	38.000	3167
Roof planks	60.00	14.00	0.073	61.25	1	38.000	2328
Metal roof	60.00	14.00	-	840.0	1	1.000	840
Beams	12.00	1.000	1.000	12.00	50	38.000	22800
Beam ties	60.00	0.667	0.333	13.33	2	38.000	1013
Bolts	1.33	0.063	-	0.0041	100	484.000	198
Rails, 4.5"	60.00	-	-	-	1	150.000	9000
Rails, 4"	60.00	-	-	-	1	110.000	6600
Total roof weight							**55980**
Total bridge weight							**101900**

The total weight of the Sulphite Bridge, including the siding that existed prior to the 1980 fire, was estimated to be 102,000 lbs, or 1.7 kips per foot of bridge length. This weight was increased by 10 percent in the computer models to account for connection hardware and other miscellaneous parts.

Dead load analysis of the Sulphite Bridge

The dead load analysis of the Sulphite Railroad Bridge provided important information on how the structure behaves under its own weight, particularly the variation in member axial stresses throughout the structure. The results of these analyses can be compared with those of the live load analyses, giving an indication of the relative importance of the dead and live loads.

Two models were used for the dead load analysis of the Sulphite Bridge (Figures 8 and 9). The weight of the bridge was distributed to each node of the truss corresponding to the contributing area. Tables 3 and 4 show the resulting element forces from the two analyses for key members.

Table 3. Element forces from dead load analysis in model with counter diagonals (negative value indicates compression)

Element	Force under dead load (kips)	Area (sq.in.)	Axial stress (psi)
Top chord U2U3	-41.142	234	-175.8
Bottom chord L2L3	41.693	207	201.4
Vertical U5L5	-17.058	200	-85.3
Tension rod U6L5	33.594	13.4	2507.0
Tension rod U4L3	3.036	3.534	1237.2
Counter diagonal U3L4	-3.818	2.454	-1080.4

Table 4. Element forces from dead load analysis in model without counter diagonals (negative value indicates compression)

Element	Force under dead load (kips)	Area (sq.in.)	Axial stress (psi)
Top chord U2U3	-43.587	234	-186.3
Bottom chord L2L3	39.050	207	188.6
Vertical U5L5	-21.480	200	-107.4
Tension rod U6L5	33.689	13.4	2514.1
Tension rod U4L3	6.408	3.534	1813.2

With the counters present and active, the top chords and verticals had slightly larger forces than when they were inactive, while the bottom chords had slightly smaller forces. Tension rod U_6L_5 did not see a significant increase in stress, only because there was no counter present in that panel to begin with, but tension rod U_4L_3 experienced a considerable increase in stress when counter U_3L_4 was not present to share the shear force in the panel. The predicted compressive force of -3.8 kips in diagonal U_3L_4 means that it would have to be prestressed with a tension equal or greater than 3.8 kips in order for it to share any of the dead load.

In the dead load analysis with counters, it was curious that U_4L_3 picked up less force than counter diagonal U_3L_4 even though U_4L_3 was the larger member. This behavior can be attributed to the deformation that occurred in the panel under the dead load, which induced more compression in the counter than tension in the main diagonal, resulting in the larger stress for counter U_3L_4.

The calculated mid-span vertical deflection in the model under dead load with counters was 0.215 inch, compared to 0.245 inch in the model without counters, a difference of about 14 percent.

Live load analysis of the Sulphite Bridge

Compared with the simple live loads used to design vehicular and pedestrian covered bridges in the late nineteenth and early twentieth century, live load analysis for railroad bridges was much more complex. Loads were applied to railroad bridge trusses at the location of engine axles, in proportion with the load distribution of the engine. For this analysis of the Sulphite Railroad Bridge, Cooper's E-50 live load, the Boston & Maine Railroad's preferred value, was used.[16]

The live load analysis was performed on the Sulphite Bridge by first drawing the influence lines for various truss elements under a 1 kip vertical load traversing the length of the truss. Using these results, Cooper's E-50 locomotive loading was moved across the model, multiplying one-half the loads (the fraction borne by each truss) by their corresponding influence line ordinates to determine the force in an element under live load. As noted above, two models were used in the analysis: the first (see Figure 8) without the counter diagonals to determine the effects without pretensioning, and the second (see Figure 9) with counters to represent the truss if pretensioned enough to make all counters active.

The main difference in these two models was the type of forces obtained in the diagonal elements of the model and their meaning in terms of design. For example, in the pretensioned truss (Figure 9), the maximum tensile force in each diagonal was computed using its respective influence line and Cooper's E-50 design live load, then the maximum compressive force was found in each diagonal, with the resulting value used to size the counter. For the unprestressed model (Figure 8), the maximum tensile and compressive forces were computed for each diagonal. The resulting compressive forces were then used to determine the pretensioning required to make all diagonals active for all live load conditions.

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[16] Kunz, F.C. *Design of Steel Bridges*, 1915, pg. 11

A comparison of the influence lines of the two models shows how the behavior of the elements is affected by the presence or absence of the counter diagonals. The following figures show the influence lines for key truss members in both situations. The two chord members show slight variations between the two models. Diagonal U_6L_5, which does not have a counter diagonal in its panel in either model, sees almost no effect from the counter diagonal in an adjacent panel. Inner diagonal U_4L_3 shows a large decrease in force when the counter diagonal is present, since the counter helps carry the shear in the panel. However, according to the model, tension rod U_4L_3 will see a stress reversal as the 1 kip vertical force moves between nodes U_3 and U_4 with or without the counter diagonal. Because of the inability of the iron rod to take compression, a sufficient pretension force must exist for it to be active. However, it is more likely that the Sulphite Bridge was not pretensioned, and the main diagonal rod usually in tension was simply assumed to carry zero force in situations where the analysis predicted compression, and the counter was added to handle, in tension, the shear force on the panel.[17]

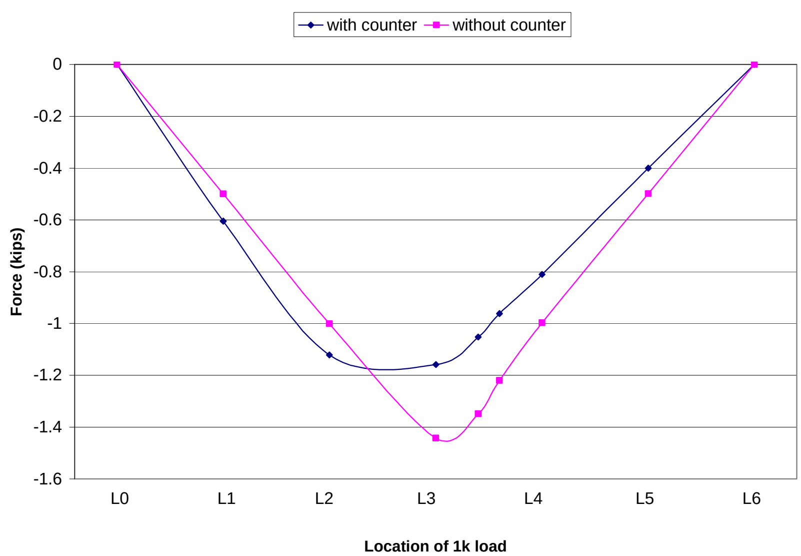

Figure 11. Influence lines for top chord U2U3 with and without counter diagonals

[17] Merriman, *Roofs and Bridges. Part I. Stresses in Simple Trusses*, 1891, pg. 53-4

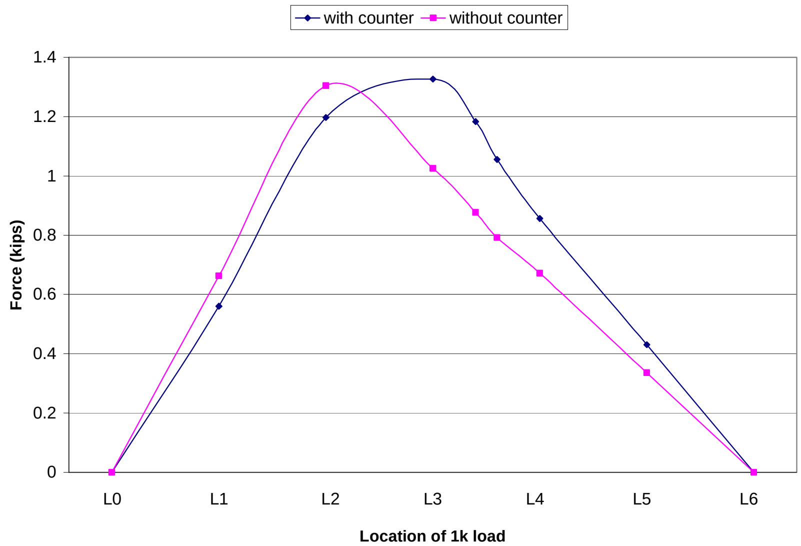

Figure 12. Influence lines for bottom chord L2L3 with and without counter diagonals

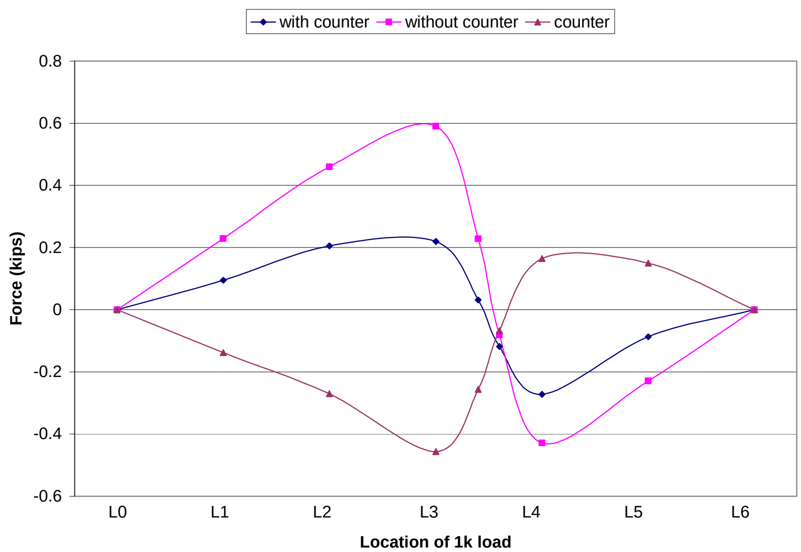

Figure 13. Influence lines for rod U4L3 with and without counter diagonals, and influence line for counter U3L4

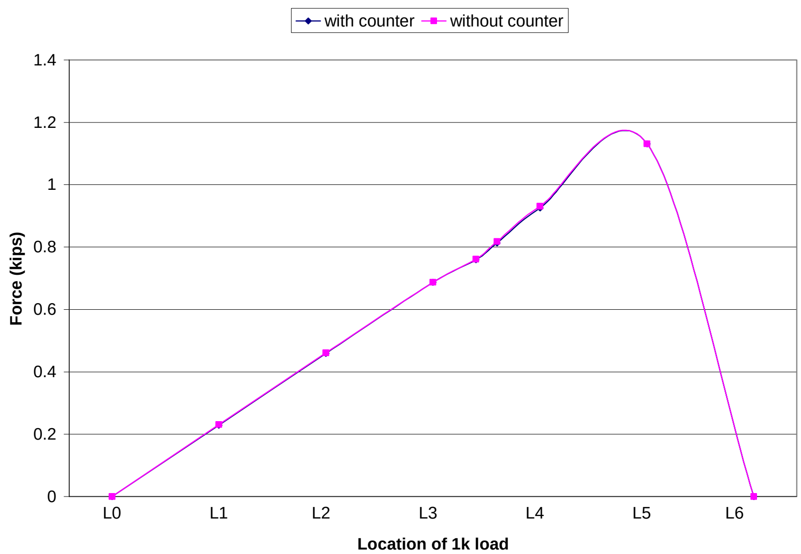

Figure 14. Influence lines for tension rod U6L5 with and without counter diagonals

Cooper's E-50 was first applied to the Sulphite Bridge model without counter diagonals. To determine the largest force induced by a train, the first 50 kip load was applied at the location of highest ordinate of the influence line, and each axle load from the train was multiplied by the corresponding force on the influence line. The second 50 kip load was then applied to the location of highest force on the influence line, then the third, and so on. Forces due to all axle loads were then totaled, and the load position that yielded the highest total force became the case for which that member was sized.

As an example of this technique, consider the top chord element U_2U_3. The influence line for U_2U_3 (Figure 11) indicates the greatest force, -1.45 kips, occurs when the 1 kip vertical load is at node U_3. Multiplying this force by 25 kips (half of one 50 kip axle load given by Cooper's E-50) gives the force in the element due to that axle of the train. Each subsequent axle load is multiplied by its corresponding force on the influence line. The total force due to a Cooper's E-50 train loading with the second 50 kip load at U_3 is 152.1 kips, or 0.650 ksi stress, for this case. Repeating this procedure for the third and fourth 50 kip axle loads at node U_3 yields a total force of 154.0 kips (0.658 ksi) and 151.8 kips (0.649 ksi) respectively. Since the third 50 kip axle load at U_3 yields the greatest stress in the element, this is the load case for which top chord U_2U_3 would be designed.

Because of the stress reversal in tension rod U_4L_3, two load cases need to be considered for the element design: the load condition that causes the greatest tensile stress, and the load condition that causes the greatest compressive stress. Loading the truss from left to right, starting with the first axle of the first engine at node U_3 (location of the largest tensile force according to the influence line), a total force of 39.3 kips (11.1 ksi stress) is found in the element. For the first 50 kip axle load of the first engine at node U_3, the force is increased to 46.9 kips (13.3 ksi), and for the second 50 kip axle load of the first engine at node U_3, the force decreases to 42.9 kips (12.1 ksi). Therefore, the worst-case load condition for tension in tensile rod U_4L_3 is when the first 50 kip axle is at node U_3. A graphical representation of these live load calculations is given in Figure 21.

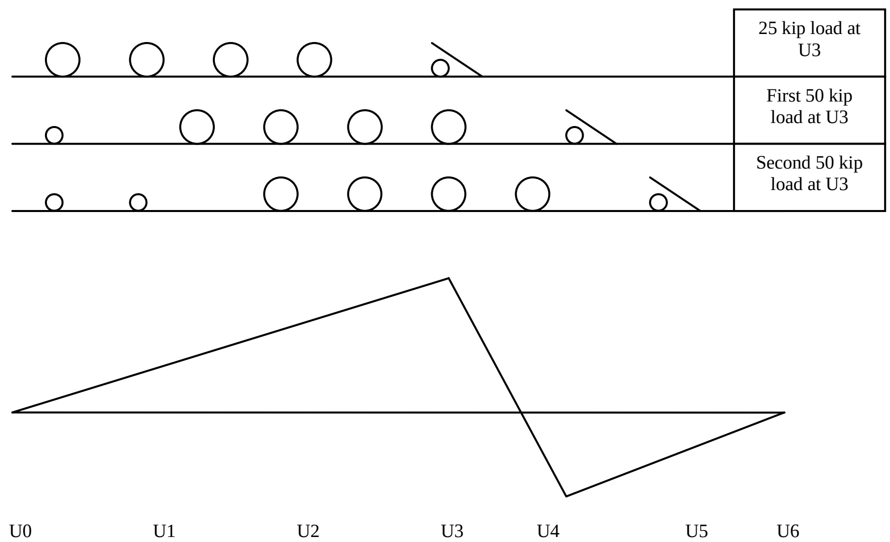

Figure 15. Live load analysis to find maximum tensile force in diagonal U4L3

The model was then loaded from right to left to find the largest compressive force in rod U_4L_3. With the front of the first train engine at node U_4, the location of the largest compressive force on the influence line, a compressive force of 16.3 kips (4.62 ksi stress) was produced in the element. Placing the first and second 50 kip axle loads of the first engine at node U_4 yielded compressive forces of 21.5 kips (6.07 ksi) and 14.6 kips (4.12 ksi), respectively. Hence, the first 50 kip axle load of the first engine at node U_4 had to be considered in the design of counter diagonal U_3L_4. A graphical representation of this process is given in Figure 16.

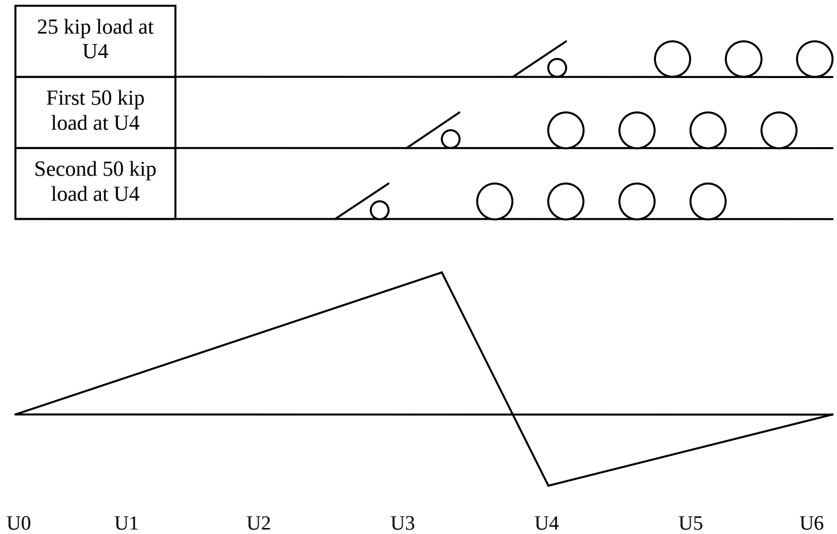

Figure 16. Live load analysis to find maximum compressive force in diagonal U4L3

For bottom chord L_2L_3, the worst load case with Cooper's E-50 loading occurred when the second 50 kip axle load was at node U_2 with the train moving from right to left. This yielded a force of 141.1 kips (0.68 ksi stress). Tension rod U_6L_5 experienced a maximum force of 124.0 kips (9.26 ksi) when the second 50 kip axle load was at node U_5.

Comparing these results with those of the dead load analyses shows the dominance of the live load in determining the required strength for this bridge. Table 5 compares the live load results calculated with Cooper's E-50 with those of the dead load analysis, both for the model of Figure 11. Maximum combined dead load and live load stresses are very close to those allowable for wood and iron.

Table 5. Comparison of dead load and live load results

Element	Dead load axial stress (ksi)	Live load axial stress (ksi)
Top chord U2U3	-0.186	-0.658
Bottom chord L2L3	0.187	0.680
Diagonal U6L5	2.51	9.26
Diagonal U4L3	1.81	13.30

Bearing stresses at nodal castings

One potential disadvantage of the Pratt truss is the tendency for the wooden chords to be crushed cross-grain at the nodal castings. To check the force bearing on the bottom chord, the maximum force exerted by diagonal U_6L_5 by both dead load and live load was calculated to find the load the nodal casting exerts on the chord perpendicular to the grain of the wood.

The nodal castings used at the bottom chord connections of the verticals are approximately 2'-9" wide by 1'-11" long, and they bear on three parallel chords of 6" width. Thus, the total bearing area for each casting is approximately the product of 3 times 6" times 23", or 414 square inches. For the dead load, a force of 33.6 kips was induced in the diagonal. Since the horizontal component of this load was assumed to be taken by the chord, only the vertical component of 23.7 kips was used. For the live load, the maximum force of 123.9 kips contributed 87.5 kips perpendicular to the chord. The sum of these forces resulted in a bearing stress of 268 psi, well within the 375 - 440 psi allowable range for pine.

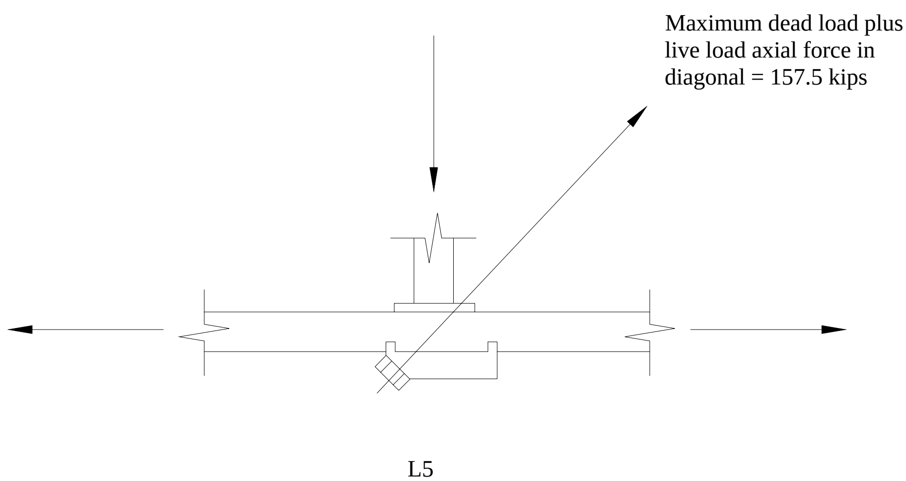

Figure 17. Bearing on lower chord due to nodal casting

The dimensions of the top nodal casting were unknown at the time of this analysis, but they were conservatively assumed to be 25 percent smaller than the bottom castings, which yielded a bearing area of 310.5 square inches. Considering the same diagonal and force results in a bearing stress of 358 psi, which is still below the allowable range of bearing stress. Therefore, these node castings should provide an adequate factor of safety against crushing of the chords.

Creep analysis of the Sulphite Bridge

Wood has time-dependent stress-strain behavior. For example, creep is the increase of strain over time in wood at a constant stress. Creep occurs in three stages. The first stage of creep deformation occurs when the structure is initially loaded, the second stage is reached when the rate of creep begins to approach zero, and the third stage, which occurs under large live loads over long periods of time, is when the rate of creep increases quickly to failure. The second stage is typically reached within a short period of time, and it can last for many years.[18]

[18] Forest Products Laboratory, *Wood Handbook, Wood as an Engineering Material*, 1999

Accurate analysis of creep requires viscous stress-strain models, however, creep behavior can be approximated with linear elastic analysis. With linear elastic models, creep strains are simulated using effective nodal loads.

Effects due to creep on the Sulphite Railroad Bridge were analyzed using the two truss models shown in Figures 8 and 9 in order to determine the behavior of the bridge under prestressed and unprestressed conditions. A creep strain of ±0.0005 was assumed. Results are given in Table 6.

Table 6. Effects of creep under assumed creep strain of ±0.0005 (negative value indicates compression)

Element	Change in element axial force due to creep strain, with counters (kips)	Change in element axial force due to creep strain, without counters (kips)
Top chords		
U0U1	1.2	1.1
U1U2	9.0	0.5
U2U3	8.4	1.3
Bottom chords		
L0L1	0.3	0.3
L1L2	7.8	-0.4
L2L3	7.2	-0.3
Verticals		
U0L0	0.6	0.6
U1L1	9.3	0.8
U2L2	16.0	0.4
U3L3	15.7	1.2
Diagonals		
U0L1	-2.09	-1.93
U1L2	-11.85	-0.10
U2L3	-11.01	-1.44
Counter diagonals		
U2L1	-11.86	-
U3L2	-11.02	-

Comparing the results of the two analyses revealed that the unprestressed, statically determinate model in Figure 8 showed very small changes in element forces due to creep. For example, the midspan vertical in the model without counter diagonals saw a tensile force due to creep of 1.2 kips, while the model with counters experienced a compressive force of 15.7 kips. These differences also occurred in the forces in the iron diagonals. Tension rod U_2U_3 experienced 1.44 kips compression as a result of creep in the model without counter diagonals, but an 11.01 kip compressive force in the model with counters.

Though a simple model was used here, the unprestressed Sulphite Bridge did not see any significant overall effect on forces due to creep. With prestressed main and counter diagonals, the calculated forces due to creep were as much as two orders of magnitude higher. It should be noted that had analyses been performed using a plane truss model, the results for the model without counter diagonals would have been negligible or zero, but the plane frame model used included bending, so some resulting element forces were computed.

To determine the overall effects of creep on the Sulphite Bridge, the results of this analysis should be compared with those of the dead load analyses. For the chords, verticals, and end-span tension rods, dead load forces were significantly larger than the forces due to creep, however, in the panels near mid-span, the tension and counter rods experienced significantly higher forces under creep in the prestressed model than they did under dead load.

In the creep analysis, the prestressed model had a downward vertical displacement of 0.61 inch, and the model without counters deflected 0.68 inch. These are almost three times larger than the dead load vertical deflections of 0.215 inch and 0.245 inch, respectively, and they serve to illustrate the large effect creep can have on displacements of wooden bridges.

Conclusions and Observations

The Sulphite Bridge is a Pratt deck truss most likely built without prestressing. As such, it could have been analyzed and designed as a statically determinate form, with counter diagonals intended to become active only when the main diagonals went into compression. The diagonals are connected by way of elaborate nodal castings that bear against the outside of the chords. The lateral bracing system of the bridge, with cross-diagonals and tightened rods, utilizes the Howe truss system. This bridge, a rare survivor of its type, is a link between the early, prestressed Pratt trusses and the single-diagonal forms that would later dominate railroad bridges, and it serves as an example of how structural designers viewed the use of counter diagonals at the time.

The Sulphite Railroad Bridge is an important piece of civil engineering history, not only for its unusual, covered-deck-truss design, but also for its use of unprestressed counters during a time of major transition in bridge design. With its careful attention to design details, such as the nodal castings, the bridge demonstrates the importance of railroads in this era and the efforts put into the design and construction of bridges. The 1980 fire that destroyed the bridge’s outer sheathing was certainly a tragedy, but, fortunately, the basic structure, though charred, still survives in a relatively sound condition. It deserves to be maintained for future generations.

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